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CHAPTER FORTY-SIX

INTERSECTIONS AT-GRADE

This Chapter discusses the geometric design of at-grade intersections. The intersection is an important part of the highway system. The operational efficiency, capacity, safety, and cost of the system depend largely upon its design, especially in an urban area. The primary objective of intersection design is to reduce potential conflicts between vehicles, bicycles, and pedestrians while providing for the convenience, ease, and comfort of those traversing the intersection.

46-1.0 GENERAL DESIGN CONTROLS

46-1.01 Design Speed

In general, the design speed for the intersection approaches should be equal to the design speed of the approach facility. However, this should not discourage the designer from using reduced geometric criteria if necessary to produce a more desirable result (e.g., to reduce driver speed prior to the intersection). If the designer can reasonably justify the design exception, the reduced criteria should be considered. To determine when reduced criteria may be applicable, the designer should consider the factors as follows:

1. heavy development along the road;
2. a posted speed limit lower than the design speed;
3. stopping sight distance;
4. adverse impacts to property owners;
5. adverse impacts to the environment;
6. low AADT;
7. construction costs;
8. adequate advance signing;
9. stop-sign control;
10. violation of driver expectancy;
11. "T" intersections;
12 short frontage roads (See Section 45-7.04); or
13. access roads with only one outlet (See Section 45-7.04).

For more information on using reduced criteria for frontage roads or local-road intersections, see Section 45-7.04.

46-1.02 Intersection Alignment

Preferably, all legs of an intersection should be on a tangent section. Where a minor road intersects a major road on a horizontal curve, the geometric design of the intersection becomes significantly more complicated, particularly for sight distance, turning movements, channelization, and superelevation. If relocation of the intersection is not practical, the minor road may be realigned to intersect the major road perpendicular to the tangent at a point on the horizontal curve. Although an improvement, this arrangement may still result in difficult turning movements if the major road is superelevated. Intersection sight distance should be considered.

Roadways should also intersect at right angles. An intersection at an acute angle is undesirable for the reasons as follows:

1. Vehicular turning movements become more restricted.
2. The accommodation of large trucks may require additional pavement and channelization.
3. The exposure time for vehicles and pedestrians crossing the main traffic flow is increased.
4. The driver's line of sight for one of the sight triangles becomes restricted.

Desirably, the angle of intersection should be within 20 deg of perpendicular. This amount of skew can often be tolerated because the impact on sight lines and turning movements may not be significant. Under restricted conditions where obtaining the right of way to straighten the angle of intersection would be impractical, an intersection angle up to 30 deg from perpendicular may be used. An intersection angle beyond these ranges may warrant more positive traffic control (all-stop or traffic signalization) or geometric improvements (realignment, greater corner sight distance). The practice of realigning roads that intersect at acute angles as shown in Figure 46-1A, Treatments for Skewed Intersections, Diagrams A and B, has proven to be beneficial. The practice of constructing short-radius curves on the side-road approaches to achieve a right-angle intersection should be avoided where practical, because it may result in increased lane encroachments.

46-1.03 Intersection Profile

The designer should avoid combinations of grade lines that make vehicular control difficult at intersections. The following criteria will apply.

46-1.03(01) Approach Gradient

The gradients of intersecting highways should be as flat as practical on those sections that will be used for storage of stopped vehicles. This is referred to as the storage platform. Desirably, the storage platform gradient should be 0.5%, not to exceed 2% where practical, on each intersecting leg within the expected storage distance on the leg (see Section 46-4.02). At a minimum, the storage platform gradient should be at least 15 m long where there are less than 10% trucks or 30 m long where there are 10% or more trucks. Gradients greater than 3% should be avoided, if practical. However, any gradient through the intersection must reflect the practicalities of matching the basic profiles of the intersecting roadways and shoulders. The intersecting roadway gradient should not exceed the grade differences (ΔG) as defined in Section 46-1.03(03) with respect to the mainline cross slope. In addition, the mainline shoulder or turn-lane cross slope and the mainline cross slope should not exceed the breakover cross slope differences shown in Figure 46-3F.

46-1.03(02) Cross Section Transitions

One or both of the approaching legs of the intersection may need to be transitioned (or warped) to meet the cross section of the two crossing roads. The designer should consider the following:

1. Stop Controlled. Where the minor road is stop controlled, the profile and cross section of the major road will normally be maintained through an intersection, and the cross slope of the stop-controlled leg will be transitioned to match the major road cross slope and profile.

2. Signalized Intersection. At a signalized or a potentially signalized intersection, the cross section of the minor road will typically be transitioned to meet the profile and cross slope of the major road. If both intersecting roads have approximately equal importance, the designer may want to consider transitioning both roadways to form a plane section through the intersection. Where compromises are necessary between the two major roadways, the smoother riding characteristics should be provided for the roadway with the higher traffic volumes and operating speeds.

3. Transition Rates. Where one or both intersecting roadways are transitioned, the designer must determine the length and rate of transition from the normal section to the modified section. See Figure 46-1B, Pavement Transitions Through Intersections. Desirably, the transition should be designed to meet the general principles of superelevation transition which apply to that roadway (i.e., open-road or low-speed urban street conditions). See Section 43-3.0 for a complete discussion on superelevation development. Where these criteria are applied to transition rates, the applied design speed is typically one of the following:

    a. 50 km/h for a stop-controlled leg;
    b. the highway design speed for a free-flowing leg; or
    c. the highway design speed for all legs of a signalized intersection.

At a minimum, the approaching legs of an intersection should be transitioned within the curb or curve radius length of the intersection consistent with practical field conditions (see Figure 46-1B, Pavement Transitions Through Intersections).

46-1.03(03) Vertical Profile

Where the cross section of the minor road is warped to meet the major road, this will result in angular breaks for traffic on the minor road if no vertical curve is inserted. If the vertical curve at the intersection cannot be designed for full stopping sight distance as discussed in Item 1 below, then lighting of the intersection should be considered. The following options are presented in order from the most desirable to the least desirable (see Figure 46-1C, Vertical Profiles of Intersecting Roads).

1. Vertical Curve (SSD). Desirably, a vertical curve should be used through an intersection which meets the criteria for stopping sight distance as described in Chapter Forty-four. For stop-controlled legs, the vertical curve should be designed to meet a design speed of 50 km/h. At free-flowing legs and at signalized intersections, the design speed of the roadway should be used to design the vertical curve.

2. Sag Vertical Curve (Comfort). For a sag vertical curve, the next most desirable option is to design the sag to meet the comfort criteria. The length of vertical curve can be determined as follows:

$$L = \frac{AV^2}{395}$$

Where:

L = length of vertical curve, m
A = algebraic difference between grades, %
V = design speed, km/h

3. Vertical Curve (Minimum Comfort). Under restricted conditions where a design based on SSD or comfort is not practical and where the design speed is 50 km/h or lower, a vertical curve at an intersection approach may be based on the formulas as follows:

$$K = (0.034V)^2 \quad \text{(Sag Curve)}$$

$$K = (0.024V)^2 \quad \text{(Crest Curve)}$$

$$L = K\,A$$

Where:

K = the horizontal distance in meters needed to produce a 1% change in the gradient along the curve
A = algebraic difference between the two tangent grades, %
V = design speed, km/h
L = length of vertical curve, m

4. Angular Breaks. Under some restricted conditions, it may be impractical to provide vertical curves on the approaches, as angular breaks are necessary through the intersection. Angular breaks may allow other intersection geometric features, such as sight distance, storage platform and drainage, to function better. Figure 46-1C, Vertical Profiles of Intersecting Roads, presents a schematic of vertical profiles through an intersection. The figure also indicates maximum angular breaks for design speeds of 50 km/h or lower. For higher design speeds, a vertical curve as discussed in Items 1 and 2 above should be used. Where angular breaks are used, the minimum chord distance between angle points should be at least 5 m.

46-1.03(04) Drainage

The profile and transitions at each intersection should be evaluated for impacts on drainage. This may require spot elevations to be shown for an intersection which may have exceptional drainage problems (e.g., an intersection which occurs in a sag vertical curve).

46-1.04 Capacity and Level of Service

The Environment, Planning and Engineering Division, in general, will perform a capacity analysis of the intersection during the preparation of the Engineer's Report. This analysis will influence several geometric design features including the number of approach lanes, lane widths, channelization, and number of departure lanes. These determinations will be based on a selected level of service and design year traffic (i.e., 20 years into the future). Level of service criteria are shown in the geometric design tables in Chapters Fifty-three through Fifty-six. Once the level of service and design traffic volumes are determined, the detailed capacity analysis is performed using the *Highway Capacity Manual* and the criteria presented in Chapter Forty-one.

46-1.05 Types of Intersections

46-1.05(01) Number of Legs

An at-grade intersection is usually a 3-leg ("T" or "Y" shape), 4-leg, or multi-leg design. An individual intersection may vary in size and shape and may be non-channelized, flared, or channelized. The principal factors which affect the selection of intersection type and its design characteristics are the DHV, turning movements, traffic character or composition, design speed, intersection angle, topography, desired type of operations, and safety.

A multi-leg intersection is that with five or more intersection legs, and should be avoided wherever practical. Where volumes are light and stop control is used, it may be satisfactory to have all intersection legs intersect at a common, all-paved area. At other than a minor intersection, safety and efficiency are improved by rearrangement that removes some conflicting movements from the major intersection. This may be accomplished by realigning one or more of the intersecting legs and combining some of the traffic movements at adjacent subsidiary intersections or, sometimes making one or more legs one-way away from the intersection.

45-1.05(02) Types of Public Road Approaches

The warrants for each type of public road approach are as follows:

1. Public Road Approach Type A. This approach should be used where the mainline shoulder is unpaved, or, if paved, is less than 2.4 m in paved width.

2. Public Road Approach Type B. This approach should be used where the mainline shoulder is paved, and is 2.4 m or wider in paved width. A paved shoulder of this width or greater will encourage use by a right-turning vehicle to clear the mainline traffic lane when decelerating for the turn.

Public road approach types A and B are designed to accommodate design vehicles WB-50 or smaller with right-hand turns beginning and ending in the traffic lanes. Right-turn lanes are not provided for these approaches. Either of these approaches should be used for a public road serving a residential, light-commercial, or light-industrial area.

3. Public Road Approach Type C. This approach should be used where the mainline shoulder is paved, is 2.4 m or wider in paved width, and an auxiliary right-turn lane along the mainline is warranted due to the right-turning traffic volume. This approach is designed to accommodate design vehicles WB-50 or smaller without encroaching onto the adjoining traffic lane. It will also accommodate a WB-65 design vehicle if a portion of the adjoining traffic lane is utilized. This approach should be used for a public road serving a residential, light-commercial, or light-industrial area.

4. Public Road Approach Type D. This approach should be used where the mainline shoulder is paved, is 2.4 m or wider in paved width, and an auxiliary right-turn lane along the mainline is warranted due to the right-turning traffic volume. This approach is designed to accommodate design vehicles WB-65 or smaller. This approach should be used where two Department-maintained routes intersect, or for a public road serving a commercial area, heavy-industrial area, or truck stop.

Figure 46-1C(1), Public Road Approach Types and Corresponding Design Vehicles, summarizes each type of public road approach and the corresponding appropriate design vehicles it can accommodate.

46-1.05(03) Determining Pavement Sections

If for a public road approach type A, B, or C, the ADT is 1000 or less, or for a public road approach type D, the ADTT of FHWA Class 5 trucks is 50 or less, the minimum pavement section shown on the INDOT *Standard Drawings* should be specified.

If for a public road approach type A, B, or C, the ADT is greater than 1000, or for a public road approach type D, the ADTT of FHWA Class 5 trucks is greater than 50, ESALs must be determined as described in *Indiana Design Manual* Section 52-8.03(01).

For an HMA approach, the required mix type is determined based on ESALs as shown in *Indiana Design Manual* Figure 52-9B. The courses and densities should be those identified in the minimum pavement section shown on the INDOT *Standard Drawings.*

For a PCCP approach, the pavement thickness is determined as described in *Indiana Design Manual* Section 52-8.03(03).

46-1.06 Intersection Spacing

When creating a new intersection, the designer must ensure that there is sufficient distance between the new and adjacent intersections so that they form distinct intersections. Short distances between intersections should be avoided, if practical, because they tend to impede traffic operations. For example, if two intersections are close together and require signalization, they may need to be considered as one intersection for signalization purposes. To operate safely, each leg of the intersection may require a separate green cycle, thereby greatly reducing the capacity for both intersections. To operate efficiently, signalized intersections should desirably be 400 m apart. In general, all new intersections should preferably be at least 120 m apart.

In addition, short gaps between opposing "T" intersections should be avoided. Drivers tend to encroach into the opposing lanes (corner cutting) so that they can turn in one movement.

46-1.07 Design Vehicles

46-1.07(01) Types

The basic design vehicles used for intersection design are as follows:

1.	P	Passenger car, light panel truck, or pickup truck
2.	SU	Single-unit truck
3.	CITY-BUS	City transit bus
4.	S-BUS-11	Conventional school bus (65 passengers)
5.	A-BUS	Articulated bus
6.	WB-12	Intermediate semitrailer combination
7.	WB-15	Intermediate semitrailer combination
8.	WB-19	Interstate semitrailer combination
9.	WB-20	(Indiana Design Vehicle) Interstate semitrailer combination
10.	WB-30T	Semitrailer combination with three trailers
11.	WB-33D	Turnpike semitrailer combination with two trailers
12.	MH	Recreational vehicle: motor home
13.	P/T	Recreational vehicle: passenger car and camper trailer
14.	P/B	Recreational vehicle: passenger car and boat trailer
15.	MH/B	Recreational vehicle: motor home and boat trailer

See Figure 46-1D, Typical Semitrailer Combination Design Vehicle illustrates a typical turning path of a semitrailer design vehicle. Section 46-12.0 provides turning templates for the design vehicles which are typically used by the Department.

46-1.07(02) Selection

In general, the selected design vehicle should be based on the largest vehicle that will use the intersection with some frequency. Figure 46-1E, Suggested Design Vehicle Selection (Intersection), identifies the desirable and minimum design vehicles based on the functional classification of the intersecting highways which the vehicle is turning from and onto.

Some portions of an intersection may be designed with one design vehicle and other portions with another vehicle. For example, it may be desirable to design physical characteristics such as curbs or islands for the IDV but to provide painted channelization markings for a passenger car. This will provide a positive indicator for the more-frequent-turning vehicle.

The SU vehicle is generally the smallest vehicle used in the design of an intersection. This reflects that, even in a residential area, delivery trucks will be negotiating turns with some frequency. On a facility accommodating regular truck traffic, one of the semitrailer combinations should be used for design. For design purposes, it can be assumed the IDV (WB-20) is permitted to operate on all public highways.

The WB-30T and WB-33D design vehicles are only permitted to operate on the Indiana Toll Road or within 25 km of its toll gates.

46-2.0 TURNING RADII (RIGHT TURNS)

Turning radii treatments for intersections at-grade are important design elements. They influence the operation, safety and construction costs of the intersection. Turning radii design may not receive sufficient attention; therefore, the designer should ensure that the design is compatible with the intersection operations. Section 46-2.01 provides detailed guidance in determining acceptable turning radii designs. Section 46-2.03 presents typical turning radii designs which may be used for preliminary design purposes.

46-2.01 Design for Pavement Edge/Curb Line

Once the designer has selected the design vehicle (Section 46-1.07), several other factors must be considered to determine the proper pavement edge/curb line. The following sections present several of the basic parameters the designer needs to consider.

46-2.01(01) Inside Clearance

Desirably, the selected design vehicle will make the right turn while maintaining approximately a 0.6-m clearance from the pavement edge or curb line and, at a minimum, will not come closer than 0.3 m.

46-2.01(02) Encroachment

To determine the acceptable encroachment, the designer should evaluate several factors including traffic volumes, one-way or two-way operations, urban/rural location and the functional classes of the intersecting roads or streets. The following will apply.

1. Urban. Desirably, the selected design vehicle will not encroach into the opposing travel lanes. However, this is not always practical or cost effective at all urban intersections. See Figure 46-2A, Guidelines for Encroachment for Right Turns (Urban Intersections). The designer must evaluate these encroachment recommendations against the construction and right-of-way impacts. For example, if these impacts are significant and if through and/or turning volumes are relatively low, the designer may decide to accept an encroachment of the design vehicle which exceeds the criteria in Figure 46-2A, Guidelines for Encroachment for Right Turns (Urban Intersections).

2. Rural. For rural intersections, the selected design vehicle should not encroach onto the adjacent lane on the road from which the turn is made nor into the opposing lanes of traffic onto the road which the turn is made.

If there are two or more lanes of traffic in the same direction on the road onto which the turn is made, the selected design vehicle can occupy both travel lanes. Desirably, the turning vehicle will be able to make the turn while remaining entirely in the right through lane.

46-2.01(03) Parking Lanes/Shoulders

At many intersections, parking lanes and/or shoulders will be available on one or both approach legs, and this additional roadway width may be carried through the intersection. This will greatly ease the turning problems for large vehicles at intersections with small curb radii. Figure 46-2B, Effect of Curb Radii and Parking on Turning Paths, illustrates the turning paths of several design vehicles where radii are 4.5 m or 7.5 m and where 2.4-m to 3.0-m parking lanes are provided. The presence of a shoulder 2.4 m to 3.0 m in width will have the same impact as a parking lane.

The figure also illustrates the necessary distance to restrict parking before the PC (4.5 m) and after the PT (6.0 to 12.0 m) on the cross street. The designer will, of course, need to check the proposed design with the applicable turning template and encroachment criteria. The designer should not consider the beneficial effects of a parking lane if the lane will be used for through traffic for part of the day.

Where there are low turning volumes, the typical shoulder pavement structure may be used. However, where there are high volumes of turns and/or where there are significant numbers of turning trucks, a full-depth shoulder pavement should be constructed. Figure 46-2B also indicates approximately where the parking lane or shoulder should have a full-depth pavement structure. This treatment is critical to avoid pavement deterioration from trucks turning at the intersection.

46-2.01(04) Pedestrians

The greater the turning radius, the farther pedestrians must walk across the roadway. This is especially important to handicapped individuals. Therefore, the designer should consider the number of pedestrians when determining the edge of pavement or curb line design. This may lead to, for example, the decision to use a simple curve with taper offsets or a turning roadway (see Section 46-3.0) to provide a pedestrian refuge.

46-2.01(05) Types of Turning Designs

Once the designer has determined the basic turning parameters (e.g., design vehicle, encroachment, inside clearance), it is necessary to select a type of turning design for the curb return or pavement edge which will meet these criteria and will fit the intersection constraints.

The simple radius is the easiest to design and construct and is often used at urban intersections. However, the simple radius with entering and exiting tapers provides a better fit to the transitional turning paths of vehicles. Therefore, the Department has determined that the simple radius with tapers should be used at all rural intersections and, desirably, at all urban intersections. A simple radius may be used for urban intersection designs. Some of the advantages of the simple radius with exiting and entering tapers as compared to the simple radius design include the following:

1. To accommodate a specific design vehicle, a radius with tapers requires less intersection pavement than a simple radius design. Another benefit is the reduced right-of-way impact at the intersection corners. For large vehicles, a simple radius is often an unreasonable design, unless a channelized island is used and, in effect, a turning roadway is installed.

3. A simple radius results in greater distances for pedestrians to cross than a radius with tapers.

4. For angles of turn greater than 90 deg, a radius with tapers is a better design than a simple radius, primarily because less intersection area is required.

46-2.01(06) Turning Template

To determine the final design, the designer must use a turning template for the selected design vehicle. The template will be applied to the intersection to determine how best to meet the criteria for turning radii design.

46-2.02 Summary

Figure 46-2C illustrates the many factors which should be evaluated in determining the proper design for right-turns at intersections. In summary, the following procedure applies:

1. Select the design vehicle (Figure 46-1E, Suggested Design Vehicle Selection (Intersections).

2. Determine the acceptable inside clearance (Section 46-2.01(01)).

3. Determine the acceptable encroachment (Section 46-2.01(02)).

4. Consider the benefits of any parking lanes or shoulders (Section 46-2.01(03)).

5. Consider impacts on pedestrians (Section 46-2.01(04)).

6. Select the type of turning treatment (Section 46-2.01(05)) (simple radius or simple radius with entering and exiting tapers).

7. Check all proposed designs with the applicable vehicular turning templates.

8. Revise design as necessary to accommodate the right-turning vehicle <u>or</u> determine that it is not practical to meet this design because of adverse impacts.

46-2.03 Typical Turning Radii Designs

Figure 46-2D, Turning Radii for Typical Design Vehicles, presents recommended minimum turning radii designs for various design vehicles, angles of turns and acceptable encroachment which may be used in the preliminary design. Figure 46-2E, Typical Turning Radii Design Assumptions, illustrates the assumptions used to develop these figures. As an alternative, the designer may want to consider using the Public Road Approach figures in the INDOT *Standard Drawings.* For the final design, the designer should check the intersection layout using the procedures presented in Section 46-2.01.

46-3.0 TURNING ROADWAYS

Turning roadways are channelized areas (separated by an island) at intersections at-grade which allow a moderate-speed, free-flowing right turn. Interchange ramps are <u>not</u> considered turning roadways.

46-3.01 Guidelines

The need for a turning roadway will be determined on a case-by-case basis. However, the following lists several guidelines the designer should consider in determining the need for a turning roadway:

1. Area Classification. Turning roadways are commonly provided in the areas as follows:

   a. Rural. In general, at the intersection of two rural arterials, turning roadways are typically provided for all right-turn movements. At the intersection of other functionally classified roadways, the need for a turning roadway will be determined on a case-by-case basis.

   b. Urban. Turning roadways are typically provided at the intersection of two urban arterials if they are within suburban and intermediate subclassifications. Because turning roadways commonly require more right-of-way than simple intersections, their use will rarely be practical in built-up areas. At the intersection of other functionally classified roadways, the need for a turning roadway will be determined on a case-by-case basis.

2. Speed. Turning roadways are desirable when the turning speed is 20 km/h or more.

3. Angle of Turn. Turning roadways should be considered when the angle of turn is greater than 90 deg. Intersections with turns less than 90 deg generally do not lend themselves to the use of a turning roadway.

4. Island Size. If there is a significant amount of unused pavement, the designer should consider using a turning roadway. Desirably, the island size should be at least 10 m^2. At a minimum, in rural areas the island should be at least 7 m^2 and in urban areas 5 m^2. If the island will provide a refuge area for pedestrians, the minimum island size should be at least 15 m^2.

5. Island Type. Islands which are 7 m^2 or greater should be constructed using a raised corrugated island and delineated with pavement markings (paint and/or raised markings) or color contrasting pavements. Islands less than 7 m^2 are typically only painted.

6. Traffic Volumes. A turning roadway should be considered if during the design hour there are 50 or more right-turning vehicles from a 2-lane facility or 100 or more right-turning vehicles from a 4-lane facility. The design hour is considered to be 20 years in the future.

7. Level of Service. Installation of a turning roadway can often improve the level of service through the intersection. At signalized intersections, a turning roadway may significantly improve the capacity of the intersection by removing the right-turning vehicles from the signal. Level-of-service criteria are provided in the geometric design tables in Chapters Fifty-three and Fifty-five.

8. Crashes. A turning roadway should be considered if there are significant numbers of rear-end type crashes at an intersection. Turning roadways allow vehicles to make the turning movements at higher speeds and, consequently, should reduce these types of accidents.

9. Pedestrians. If pedestrian volumes are high, a turning roadway provides a refuge area for pedestrians crossing a wide intersection.

10. Trucks. Turning roadways should be considered when the selected design vehicle is a semi-trailer combination.

11. Width. Generally, the turning roadway width should not be less than 4.2 m.

Figure 46-3A, Typical Turning Roadway (Stop Controlled on Minor Road), illustrates a typical design for a turning roadway. The figure presents a turning roadway with a simple curve radius with entering and exiting tapers. Turning roadways with a simple curve design are also acceptable.

46-3.02 Design Criteria

46-3.02(01) Design Speed

Desirably, the design speed on a turning roadway should be within 30 km/h of the mainline design speed. However, a turning roadway even at a low design speed (e.g., 20 km/h) will still provide a significant benefit to the turning vehicle regardless of the speed on the approaching highway. Typically, the design speed for a turning roadway will be in the range of 20-30 km/h.

46-3.02(02) Width

Turning roadway widths are dependent upon the turning radii and design vehicle selected. Figure 46-1E, Suggested Design Vehicle Selection (Intersections), provides the criteria for selection of the appropriate design vehicle. Figure 46-3B, Derived Pavement Widths, m, for Turning Roadways for Different Design Vehicles, presents the turning roadway pavement widths for various design vehicles based on 1-lane, one-way operation with no provision for passing a stalled vehicle. This design is generally appropriate for most at-grade intersections. The pavement widths in Figure 46-3B provide an extra 1.8-m clearance beyond the design vehicle's swept path. This additional width provides extra room for maneuverability and driver variances.

In selecting the turning roadway width, the designer should also consider the possibility that a larger vehicle may also use the turning roadway. To some extent, the extra 1.8-m clearances in Figure 46-3B will allow for the accommodation of the occasional larger vehicle, although at a lower speed and with less clearance. For example, a turning roadway design for a WB-15 with a 30-m radius will still accommodate an occasional WB-19 vehicle. However, it would not accommodate a WB-20 vehicle. If there are a significant number of the larger vehicle using the turning roadway, it should be selected as the design vehicle.

A shoulder is typically provided on the right side of a right-turning roadway in rural areas. Desirably, the width of the shoulder should be the same as the preceding mainline shoulder. However, at restricted intersections, a narrower or no shoulder may be provided. Where shoulders are provided, a full-depth shoulder pavement should be constructed.

Additional information on turning roadway widths can be found in AASHTO's *A Policy on Geometric Design of Highways and Streets* (One-lane, one-way operation with provision for passing a stalled vehicle by another of the same type, and two-lane operation).

46-3.02(03) Pavement Thickness

The entire turning roadway width, including shoulders and curb offsets, should have a uniform pavement thickness. See Chapter Fifty-two for additional information on pavement designs.

46-3.02(04) Horizontal Alignment

The horizontal alignment of turning roadway design differs from that of open-roadway conditions, which are discussed in Chapter Forty-three. In comparison, turning roadway designs are less restrictive, which reflects more restrictive field conditions and less demanding driver expectation and driver acceptance of design limitations. The following discusses several of the assumptions used to design horizontal alignment for turning roadways:

1. <u>Curvature Arrangement</u>. For most turning roadway designs, a simple curve with an entrance and exit taper is the typical curvature arrangement.

2. <u>Superelevation</u>. Turning roadways are often relatively short in length. This greatly increases the difficulty of superelevating the roadway. Therefore, a flexible approach is used for superelevating turning roadways. Figure 46-3C, Superelevation Rates (Turning Roadways), provides a range of superelevation rates that the designer may select for various combinations of curve radii and design speeds. For many turning roadways with low design speeds (e.g., 20-30 km/h), the superelevation rate will typically be 2%, the normal cross slope. The maximum superelevation rate for turning roadways should not exceed 6%. Selection of the appropriate superelevation rate will be based on field conditions and will be determined on a site-by-site basis.

3. <u>Superelevation Transitions</u>. Desirably, when a turning roadway is superelevated, the transition length should meet the criteria presented in Chapter Forty-three for the relative longitudinal slope. For open roadways, the relative slope is measured between the centerline of the roadway and either pavement edge. The relative longitudinal slope is measured between the left edge of the turning roadway and the right pavement edge. For turning roadways, the axis of rotation is normally about the left edge of the traveled way.

   Due to the restrictive nature of turning roadways and their typically short lengths, minimum transition lengths will be determined on a case-by-case basis. The designer should review the field conditions, deceleration and acceleration taper lengths, right-of-way restrictions and construction costs to produce a practical design for superelevation transition lengths at turning roadways.

4. <u>Superelevation Development</u>. See Figure 46-3D, Development of Superelevation at Turning Roadway Terminals. The actual development will depend upon the practical field conditions combined with a reasonable consideration of the theory behind horizontal curvature. The following presents criteria which should be met:

   a. No change in the normal cross slope is necessary up to Section B-B. Here, the width of the turning roadway is about 0.6 m.

   b. The full width of the turning roadway should be attained at Section D-D. The amount of superelevation at D-D will depend upon the practical field conditions.

   c. Beyond Section D-D, the turning roadway pavement should be rotated as needed to provide the required superelevation for the design speed of the turning roadway.

   d. The minimum superelevation transition lengths should meet the criteria set forth in Item 3.

   e. The superelevation treatment for the exiting portion of the turning roadway should be similar to that described for the entering portion. However, for stop-control merges the superelevation on the turning roadway should match the cross slope on the merging highway or street.

   See the associated discussion shown in the AASTHO *Policy on Geometric Design of Highways and Streets* for more information regarding the specific situations as follows:

   a. Turning roadway leaves a through road that is on tangent.

   b. Turning roadway and through lanes curve in same direction.

c. Turning roadway and through lanes curve in opposite directions.

d. There is a speed-change lane.

5. Minimum Radius. The minimum turning roadway radii are based on design speed, side-friction factors and superelevation (see Chapter Forty-three). Figure 46-3E, Typical Designs for Turning Roadways, presents minimum radii for various turning roadway conditions. As discussed in Item 2, a range of superelevation rates may be used. Therefore, Figure 46-3E presents minimum radii for several assumed superelevation rates.

6. Cross Slope Rollover. Figure 46-3F, Pavement Cross Slope at Turning Roadway Terminals, presents the maximum allowable algebraic difference in the cross slopes between the mainline and turning roadway where these two are adjacent to each other. In Figure 46-3D, these criteria apply between Section A-A and Section D-D. This will likely be a factor only when a superelevated mainline is curving to the left.

7. Stopping Sight Distance. The values for stopping sight distance for open highway conditions are applicable to turning roadway intersections of the same design speed. The values shown in Figure 42-1A, together with the value for a design speed of 15 km/h, are shown below in Figure 46-3E_1, Stopping Sight Distance for Turning Roadways.

   The sight distances should be available at all points along a turning roadway. Wherever practical, longer sight distances should be provided. They apply as controls in design of both horizontal and vertical alignment.

   For design speeds of less than 60 km/h, sag vertical curves, as governed by headlight sight distances, theoretically should be longer than crest vertical curves. Because the design speed of most turning roadways is governed by the horizontal curvature, and the curvature is relatively sharp, a headlight beam parallel to the longitudinal axis of the vehicle ceases to be a control. Where practical longer lengths for both crest- and sag vertical curves should be used.

   The sight distance control as applied to horizontal alignment has an equal, if not greater effect on design of turning roadways than vertical control. The sight line along the centerline of the inside lane around the curve, clear of obstructions, should be such that the sight distance measured on an arc along the vehicle path equals or exceeds the stopping sight distance shown in Figure 46-3E_1. A likely obstruction may be a bridge abutment or line of columns, wall, cut sideslope, or a side or corner of a building.

46-3.02(05) Deceleration/Acceleration Lanes

Deceleration/acceleration lanes are often desirable when turning roadways are used. However, they may not always be practical when considering field conditions, right-of-way restrictions and construction costs. The following offers several guidelines for the designer to consider in determining the need for a deceleration or acceleration lane with a turning roadway:

1. Turning Roadway Design Speed. The use of deceleration and/or acceleration lanes should be considered where the turning roadway design speed is more than 30 km/h lower than that of the mainline design speed.

2. Mainline Design Speed. Deceleration and/or acceleration lanes should be considered if the mainline design speed is 80 km/h or greater.

3. Traffic Volumes. An acceleration and/or deceleration lane should be considered where the following conditions exist:

    a. Two-Lane Facility. Acceleration and/or deceleration lanes should be considered where the mainline traffic volume is 5000 vpd or more and there are 75 or more turning vehicles during the design peak hour.

    b. Four-Lane Facility. Acceleration and/or deceleration lanes should be considered where the mainline traffic volume is 10,000 vpd or more and there are 125 or more turning vehicles during the design peak hour.

4. Storage Length. Deceleration lanes may be beneficial at signalized intersections where the through lane storage may limit the access to the turning roadway. In these cases, the designer should consider a deceleration lane which extends upstream beyond the storage requirements of the intersection to allow access for right-turning vehicles into the turning roadway (see Figure 46-3G, Additional Length of Turning Roadway (Signalized Intersection)).

5. Traffic Condition. An acceleration lane should be provided if the merge traffic condition is free flowing. Acceleration lanes are generally not considered for yield or stop control conditions.

The length of deceleration and acceleration lanes are based on the design speed of the turning roadway and the design speed of the mainline. Desirably, deceleration and acceleration lengths should meet the criteria in Section 48-4.0 for ramps at interchanges.

46-3.02(06) Pavement Markings

See Figure 46-3H, Typical Pavement Markings for Turning Roadways. For additional information on pavement markings, the designer should review Chapter Seventy-six.

46-4.0 RIGHT- AND LEFT-TURN LANES

When the turning maneuver for left- and right-turning vehicles occurs in the through travel lanes, it disrupts the flow of through traffic. To minimize potential conflicts, the use of turn lanes may be warranted for at-grade intersections to improve the level of service and safety at the intersection.

46-4.01 Turn Lane Warrants

46-4.01(01) Warrants for Right-Turn Lanes

The use of right-turn lanes at intersections can significantly improve operations. Exclusive right-turn lanes should be considered as follows:

1. at any unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria in Figure 46-4A, Guidelines for Right-Turn Lanes at Unsignalized Intersections on 2-Lane Highways;

2. at any unsignalized intersection on a high-speed 4-lane urban or rural highway which satisfies the criteria in Figure 46-4B, Guidelines for Right-Turn Lanes at Unsignalized Intersection on 4-Lane Highways;

3. at any intersection where a capacity analysis determines a right-turn lane is necessary to meet the level-of-service criteria;

4. for uniformity of intersection design along the highway if other intersections have right-turn lanes; or

5. at any intersection where the accident experience, existing traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to right-turning vehicles.

46-4.01(02) Warrants for Left-Turn Lanes

The accommodation of left turns is often the critical factor in proper intersection and median openings designs. Left-turn lanes can significantly improve both the level of service and intersection safety. Exclusive left-turn lanes should be provided as follows:

1. at each intersection on an arterial, where practical;

2. at each intersection on a divided urban or rural highway with a median wide enough to accommodate a left-turn lane, provided that adequate spacing exists between intersections;

3. at any unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria in Figure 46-4C, Volume Guidelines for Left-Turn Lanes on Two-Lane Highways;

4. at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria, including multiple left-turn lanes;

5. at any signalized intersection where the design hour left-turning volume is 60 veh/h or more for a single turn lane, or where a capacity analysis determines the need for left-turn lanes;

6. for uniformity of intersection design along the highway if other intersections have left-turn lanes in order to satisfy driver expectancy;

7. at any intersection where the accident experience, traffic operations, or sight distance restrictions (e.g., intersection beyond a crest vertical curve) present, or engineering judgment perceives, significant conflicts related to left-turning vehicles; or

8. at a median opening of sufficient median width where at least 100 left turns are made during the design hour, or where the vehicular speeds are greater than or equal to 80 km/h.

<u>46-4.02 Design of Left- and Right-Turn Lanes</u>

46-4.02(01) Turn Lane Width

The width of the turn lane should be determined relative to the functional class, urban/rural location, and project scope of work. Chapters Fifty-three and Fifty-five provide the applicable widths for auxiliary lanes. In addition, those chapters provide criteria for the applicable shoulder width adjacent to an auxiliary lane.

46-4.02(02) Turn Lane Length

Desirably, the length of a right- or left-turn lane at an intersection should allow both safe vehicular deceleration and storage of turning vehicles outside of the through lanes. However, it is often not practical to provide a turn lane length which provides for deceleration. Therefore, the full-width length will often only be sufficient for storage.

The length of an auxiliary lane will be determined by some combination of its taper length, L_T, deceleration length, L_D, and storage length, L_S, and by the mainline functional classification. Figure 46-4H, Functional Lengths of Auxiliary Turning Lanes, provides the length considerations for the various classifications. See Figure 46-4 I, Typical Auxiliary Lanes at an Intersection. The following will apply.

1. Taper. For tangent approaches, the Department's practice is to use a 30-m straight-line taper at the beginning of a single turn lane, or a 45-m straight-line taper at the beginning of dual turn lanes for an urban street. On a curvilinear alignment, the entrance taper should typically be designed with a constant rate of divergence throughout the curve. The entrance taper length should be at least 15 m.

2. Deceleration. For a rural facility, the deceleration distance, L_D, should meet the criteria shown in Figure 46-4J, Deceleration Distances for Turning Lanes. In addition, the values determined from Figure 46-4J should be adjusted for grades. Figure 46-4J also provides these grade adjustment factors. These distances are desirable on an urban facility; however, this is not always feasible. Under restricted urban conditions, deceleration may have to be accomplished entirely within the travel lane. For this situation, the length of turn lane will be determined solely on the basis of providing adequate vehicle storage, i.e., $L_D = 0$ m.

3. Storage Length (Signalized Intersection). The storage length, L_S, for a turn lane should be sufficient to store the number of vehicles likely to accumulate in a signal cycle during the design hour. The following should be considered in determining the recommended storage length for a signalized intersection.

    a. The storage length should be based on the cycle length and the traffic volumes during the design hour. For a cycle of less than 120 s, the storage length should be based on 2 times the average number of vehicles that would store during the cycle during the design hour. For a cycle of 120 s or longer, the storage length should be based on 1.5 times the average number of vehicles that would store during the cycle during the design hour. Average vehicle length is assumed to be 6.1 m. At a minimum, space should be provided for two passenger cars.

b. Figure 46-4K(1), Recommended Storage Length for Signalized Intersection, illustrates an alternative method to determine the recommended storage length for a left-turn lane, or a right-turn lane where a turn on red is prohibited, for a signalized intersection for which the *v/c* ratio is known. The values obtained from the figure are for a cycle length of 75 s and a *v/c* ratio of 0.80. For other values, the length obtained in the figure should be multiplied by the appropriate adjustment factor shown in Figure 46-4K, Storage Length Adjustment Factors. The *v/c* ratio is determined by a capacity analysis as described in the *Highway Capacity Manual*.

c. Where a turn on red is permitted or where a separate right-turn signal phase is provided, the length of the right-turn lane may be reduced due to less accumulation of turning vehicles.

4. Storage Length (Unsignalized Intersection). The storage length should be sufficient to avoid the possibility of a left-turning vehicle stopping in the through lanes and waiting for a gap in the opposing traffic flow. The minimum storage length should have sufficient length to accommodate the expected number of turning vehicles likely to arrive in an average 2-minute period within the design hour. At a minimum, space should be provided for two passenger cars. If truck traffic exceeds 10%, space should be provided for at least one passenger car and one truck. See Figure 46-4L, Recommended Storage Lengths (L_S) for Unsignalized Intersection.

5. Minimum Turn-Lane Length. Under restricted conditions, the minimum full-width right- or left-turn lane length, including deceleration and storage, may be 15 m where there are less than 10% trucks, or 30 m where there are 10% or more trucks. This is exclusive of the taper. See Item 1 for minimum taper length.

   At a signalized intersection, the right- or left-turn lane length should exceed the storage length of the adjacent through lane. Otherwise, a vehicular queue in the through lane will block entry into the turn lane for turning vehicles.

46-4.02(03) Channelized Left-Turn Lane

If a left-turn lane is required on a 2-lane highway, it should desirably be designed as a channelized left-turn lane as illustrated in Figure 46-4M, Channelized Left-Turn Lane for 2-Lane Highway. As an alternative, based on site conditions and turning volumes, a passing blister may be used at a T intersection. See Section 46-4.03.

46-4.02(04) Slotted Left-Turn Lane

On 4-lane facilities with wide divided medians, slotted left-turn lanes are desirable where the median width is equal to or greater than 7.3 m. The advantages are as follows:

1. better visability of opposing through traffic;
2. decreased possibility of conflict between opposing left-turning vehicles; and
3. more left-turning vehicles are served.

Figure 46-4N, Typical Slotted Tapered Left-Turn Lane (Signalized Intersection), and Figure 46-4N_1, Typical Slotted Parallel Left-Turn Lane (Signalized Intersection), illustrate typical tapered and parallel slotted left-turn lanes. In addition, the designer should consider the following:

1. Slotted Length. The slotted section of the turn lane should be at least 15 m long with a desirable minimum of 30 m. The slotted section should not include any of the required deceleration distance for the turn lane.
2. Nose Width. The nose of the slotted lane should be a minimum of 1.2 m plus any shoulder/curb offset width (and/or return taper) from the opposing through lanes. The nose position should be checked for interference with the turn paths from the cross street.
3. Slot Angle. The angle of the slot should not diverge more than 10^{o} from the through mainline alignment.
4. Island. To delineate the slotted portion, the channelized island for the slotted lane should be a raised corrugated island. Raised pavement markers may be used for further delineation.

46-4.02(05) Turn Lane Extensions

On 2-lane highways, it may be desirable to extend the right-turn lane, if provided, beyond the intersection at some four-legged intersections to allow mainline vehicles to by-pass left-turning vehicles on the right. See Section 46-4.03 for passing blister designs at three-legged intersections. When determining the need for turn lane extensions, the designer should consider the following:

1. Traffic Volumes. Turn lane extensions may be provided at the intersection of all public roads and streets on 2-lane State highways with a design year ADT of 5000 or greater.

On 2-lane State highways with a design year ADT less than 5000, turn lane extensions are only recommended if one or more of the following criteria is met:

a. there is an existing turn lane extension;

b. there are 20 or more left-turning vehicles during the design hour;

c. accident reports or site evidence, such as skid marks in the through lane displaying emergency braking, indicate potential problems with left-turning vehicles; or

d. shoulders indicate heavy use (e.g., dropped shoulders, severe pavement distress).

2. Scope of Work. Turn lane extensions should only be used in conjunction with 3R and resurfacing projects. For new and reconstruction projects, a channelized left-turn lane should be provided; see Figure 46-4M, Channelized Left-Turn Lane for 2-Lane Highway.

3. Right-Turn Lane Warrant. A turn lane extension may be appropriate at a four-legged intersection even if a right-turn lane is not warranted.

4. Design. When designing a turn lane extension, the designer should consider the following:

a. Geometrics. The beginning of the turn lane should be designed as a right-turn lane including width and length (taper, deceleration and storage). The extension beyond the intersection should be designed as a 90-m tapered acceleration lane; see Section 46-6.0. Under restricted conditions, the turn lane extension length may be shortened to meet field conditions, but not less than 60 m.

b. Pavement. The turn lane and turn-lane extension should have the same color and pavement texture as the through lanes. Shoulders adjacent to the turn lane extension should be of contrasting color and texture. The turn lane extension pavement should be a full-depth shoulder with an additional 60 m of full-depth shoulders provided after the exiting tapers.

c. Sight Distance. Decision sight distance should be provided on the mainline to the intersection to allow the mainline driver enough time to consider whether to pass the left-turning vehicle or come to a stop. Sufficient sight distance should be available so that the side street driver will not encroach into the auxiliary lane. Stop bars should be provided to delineate the proper location for stopping.

d. Concerns. Extra consideration should be given at offset intersections to ensure the turn lane extension will not lead to operational problems. Distractions such as location of driveways, commercial background lighting, highway light luminaires should also be considered in the design.

5. Channelized Left-turn Lanes. The decision on whether to use either a channelized left-turn lane or a turn lane extension will be based on accident history, right-of-way availability, through and turning traffic volumes, design speed and available sight distance. A channelized left-turn lane should be provided if the left-turn volumes are high enough that a left-turn lane is warranted as discussed in Section 46-4.01.

46-4.03 Guidelines for Passing Blisters

At some three-legged intersections, it may be desirable to provide a passing blister to relieve congestion due to left-turning vehicles. The designer should review the following when determining the need for passing blisters.

1. Traffic Volumes. Passing blisters may be provided at the intersection of all public roads and streets on two-lane INDOT routes with a design year ADT of 5000 or greater. On two-lane INDOT routes with a design year ADT less than 5000, passing blisters are only recommended if one or more of the criteria are met as follows:

   a. there is an existing passing blister;

   b. there are 20 or more left-turning vehicles during the design hour;

   c. accident reports or site evidence, such as skid marks in the through lane displaying emergency braking, indicate potential problems with left-turning vehicles; or

   d. shoulders indicate heavy use (e.g., dropped shoulders, severe pavement distress).

2. Design. Figure 46-4 O, Typical Passing Blister for a 2-Lane Highway, illustrates and provides the design criteria for passing blisters. Alternative designs should be considered if successive passing blisters overlap each other or are within close proximity to each other.

3. Channelized Left-turn Lanes. The decision on whether to use either a channelized left-turn lane or a passing blister will be based on accident history, right-of-way availability, through and turning traffic volumes, design speed and available sight distance. A

channelized left-turn lane should be provided if the left-turn volumes are high enough that a left-turn lane is warranted as discussed in Section 46-4.01.

<u>46-4.04 Multiple Turn Lanes</u>

46-4.04(01) Warrants

Multiple right- and/or left-turn lanes should be considered as follows:

1. there is insufficient space to provide the necessary length of a single turn lane because of restrictive site conditions (e.g., closely spaced intersections); and/or

2. based on a capacity analysis, the necessary time for a protected left-turn phase for a single lane becomes unattainable to meet the level-of-service criteria (average delay per vehicle).

Dual right-turn lanes do not work as well as dual left-turn lanes because of the more restrictive turning movements for two abreast right turns. If practical, the designer should find an alternative means to accommodate the high number of right-turning vehicles. For example, a turning roadway may be more efficient.

At intersections with very high-turning volumes, dual right- and/or triple left-turn lanes may be considered. However, multiple turn lanes may cause problems with right-of-way, lane alignment, crossing pedestrians and lane confusion for approaching drivers. Therefore, if practical, the designer should consider alternative designs, such as indirect left turns or an interchange.

46-4.04(02) Design

For multiple turn lanes to work properly, several design elements must be carefully evaluated. Figure 46-4P, Schematic For Multiple Turn Lanes, presents both multiple right- and left-turn lanes to illustrate the more important design elements. The designer should consider the following:

1. <u>Throat Width</u>. Because of the off-tracking characteristics of turning vehicles, the normal width of two travel lanes may be inadequate to properly receive two vehicles turning abreast. Therefore, the receiving throat width may need to be adjusted. The throat width will be determined by the application of the turning templates for the design vehicles (see Item 4).

2. <u>Special Pavement Markings</u>. As illustrated in Figure 46-4P, pavement markings can effectively guide two lines of vehicles turning abreast. The Traffic Design Section will determine the selection and placement of any special pavement markings.

3. <u>Opposing Left-Turn Traffic</u>. If simultaneous, opposing multiple left turns will be allowed, the designer should ensure that there is sufficient space for all turning movements. Desirably, this separation should be 9 m (See Figure 46-4P). This is a factor at all signalized intersections, but dual left-turn lanes with their two-abreast vehicles can cause problems. Dual turning lanes should only be used with signalization providing a separate turning phase.

4. <u>Turning Templates</u>. All intersection design elements for multiple turn lanes must be checked by using the applicable turning templates. The designer should assume that the selected design vehicle will turn from the outside lane of the multiple turn lanes. Desirably, the inside vehicle should be a SU but, as a minimum, the other vehicle can be assumed to be a passenger vehicle turning side by side with the selected design vehicle.

46-5.0 TWO-WAY LEFT-TURN LANES (TWLTL)

Two-way left-turn lanes (TWLTL) are a cost-effective method to accommodate a continuous left-turn demand and to reduce delay and accidents. These lanes will often improve operations on roadways which were originally intended to serve the through movement but now must accommodate the demand for accessibility created by changes in adjacent land use.

<u>46-5.01 Guidelines</u>

The following provides general guidelines for where the TWLTL should be considered.

1. <u>General</u>. The physical conditions under which a TWLTL should be considered include the following:

   a. areas with a high number of driveways per km (e.g., 30 driveways total per km on both sides);

   b. areas of high-density commercial development; and/or

   c. areas with substantial mid-block left turns.

   The applicability of the TWLTL is a function of the traffic conditions resulting from the adjacent land use. The designer should evaluate the area to determine the relative attractiveness of a TWLTL as compared to alternative access techniques. For example, a TWLTL may perpetuate more strip development. If this is not desirable, then a raised median may be more appropriate.

2. Functional Class. Undivided 2-lane and 4-lane urban or suburban arterials are the most common candidates for the implementation of a TWLTL. These are commonly referred to as 3-lane and 5-lane facilities, respectively. The use of a TWLTL on a 6-lane arterial (i.e., a 7-lane facility) is generally not appropriate. See Section 46-5.03.

3. Traffic Volumes. Traffic volumes are a significant factor in the consideration of a TWLTL. The design year which should be used to determine the traffic volumes is 20 years. As general guidance, the following should be used:

    a. On existing 2-lane roadways, a TWLTL will often be advantageous for traffic volumes between 5,000 and 12,500 ADT.

    b. On existing 4-lane highways, a TWLTL will often be advantageous for traffic volumes between 10,000 and 25,000 ADT.

    c. For traffic volumes greater than 25,000 ADT, a raised median may be more appropriate. For a 6-lane highway, a raised median is recommended.

    d. Pedestrian crossing volumes are also a consideration because of the large paved area which must be traversed when a TWLTL is present (i.e., no pedestrian refuge exists).

4. Speed. The design speed on a highway facility is a major factor in TWLTL applications. Experience indicates that design speeds from 40 km/h to 80 km/h will properly accommodate a TWLTL. For posted speeds higher than 80 km/h, their use should be considered only on a case-by-case basis.

5. Accident History. On high-volume urban or suburban arterials, traffic conflicts often result because of a significant number of mid-block left turns combined with significant opposing traffic volumes. This may lead to a disproportionate number of mid-block, rear-end and/or sideswipe accidents. A TWLTL is likely to reduce these types of accidents. The designer should review and evaluate the available accident data to determine if unusually high numbers of these accidents are occurring.

46-5.02 Design Criteria

46-5.02(01) Lane Width

Recommended lane widths for a TWLTL are presented in Chapters Fifty-three and Fifty-five. Existing highways that warrant the installation of a TWLTL are often located in areas of restricted right-of-way, and conversion of the existing cross section may be difficult. To obtain the TWLTL width, the designer may have to consider several alternatives including the following:

1. removing an existing raised median,
2. reducing the width of existing through lanes,
3. reducing the number of existing through lanes,
4. eliminating existing parking lanes,
5. eliminating or reducing the width of existing shoulders, and/or
6. acquiring additional right-of-way to expand the pavement width by the amount needed for the TWLTL.

Desirably, Item 1 or 6 would be the most advantageous alternative. If this is not practical, the designer will have to seriously evaluate the trade-offs between the benefits of the TWLTL and the negative impacts of eliminating or reducing the width of the existing cross section elements. This may involve a capacity analysis or an in-depth evaluation of the existing accident history.

46-5.02(02) Intersection Treatment

At all intersections with public roads, the TWLTL must either be terminated in advance of the intersection to allow the development of an exclusive left-turn lane or be extended up to the intersection. In most cases where the TWLTL is extended up to the intersection, the pavement marking will switch from two opposing left-turn arrows to one left-turn arrow only. When determining the intersection treatment, the following should be considered.

1. <u>Signals</u>. At signalized intersections the TWLTL should be terminated because these intersections will typically warrant an exclusive left-turn lane. At unsignalized intersections, the TWLTL may be extended through the intersection because an exclusive left-turn lane is usually not required.
2. <u>Turning Volumes</u>. The left-turn demand into the intersecting road is a factor in determining the proper intersection treatment. As general guidance, if the minimum

storage length will govern (See Section 46-4.02), it will probably be preferable to extend the TWLTL up to the intersection (i.e., provide no exclusive left-turn lane).

3. Minimum Length of TWLTL. The TWLTL should have sufficient length to operate properly, and the type of intersection treatment will determine the length of the TWLTL. The appropriate minimum length is influenced by the operating speeds on the highway. The following guidance may be used.

   a. On facilities where $V \leq 50$ km/h, the minimum uninterrupted length of a TWLTL should be 150 to 300 m.

   b. On facilities where $V > 50$ km/h, the minimum uninterrupted length of a TWLTL should be at least 300 m.

   The final decision on the length of the TWLTL will be based on site conditions.

4. Operational/Safety Factors. Extending the TWLTL up to an intersection could result in operational or safety problems. Some drivers may, for example, pass through the intersection in the TWLTL and turn left just beyond the intersection into a driveway which is very close to the intersection (e.g., within 10 m). If operational or safety problems are known or anticipated at an intersection, it may be preferable to remove the TWLTL prior to the intersection (i.e., provide an exclusive left-turn lane).

5. Pavement Markings. Figure 46-5A, Typical Pavement Markings for a TWLTL, illustrates markings at an unsignalized and a signalized intersection. Chapter Seventy-six provides additional information for marking TWLTL.

46-5.03 6-Lane Sections

For traffic volumes greater than 25,000 ADT, the designer should use a 6-lane section with a raised median. The decision on whether to provide a TWLTL instead of a raised median will be determined on a case-by-case basis. The following lists several factors the designer should consider in making this determination:

1. TWLTLs tend to be safer when the following conditions exist, or are proposed.

   a. at least 45 driveways per kilometer;

   b. fewer than 1 signalized intersection per kilometer; and

c. a maximum of 3 to 4 approaches per kilometer, depending on the number of signals per kilometer.

2. There may be insufficient gaps available in the oncoming traffic to allow a vehicle to make a left turn from the TWLTL in an acceptable period of time.

3. Left-turning vehicles from roadside driveways may try to use the TWLTL as an acceleration lane or as a waiting area before merging in with the mainline traffic.

4. There may be significant delays at signals to handle crossing pedestrians with a TWLTL. A raised median may be able to provide a refuge area for crossing pedestrians.

5. At signalized intersections requiring dual-turn lanes, it will be more difficult to develop the additional lane with TWLTL. In addition, there may be more lane confusion at the intersection for the approaching driver.

6. Raised medians force drivers to make all their left turns at the intersections, which may overload the capacity of the intersection and increase driver travel time.

7. With raised medians, the left-turn movements are concentrated at the intersections, thereby reducing the conflict area of the overall facility.

8. Raised medians may discourage patrons from using facilities on the other side (e.g., gas stations, convenience stores, restaurants).

9. Raised median sections discourage new strip development, where as TWLTL's may encourage such development.

46-6.0 INTERSECTION ACCELERATION LANES

It may be necessary to provide an acceleration lane for turning vehicles at an intersection to allow these vehicles to accelerate before merging with the through traffic.

<u>46-6.01 Acceleration Lanes for Right-Turning Vehicles (Guidelines)</u>

The following provides general guidelines for the consideration of an acceleration lane for right-turning vehicles.

1. where the intersection is near or at capacity (LOS E) in the design year;

2. where a turning roadway is used (see Section 46-3.0);

3. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility;

4. where there is a significant history of rear-end and/or sideswipe accidents;

5. where there is inadequate intersection sight distance available; and/or

6. where there are high volumes of trucks turning onto the mainline.

46-6.02 Acceleration Lanes in Medians (Guidelines)

The following provides general guidelines for the consideration of an acceleration lane in the median for left-turning vehicles.

1. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility. The acceleration lane may reduce the need for a signalized intersection;

2. where there is a significant history of rear-end and/or sideswipe accidents;

3. where there is inadequate intersection sight distance available; and/or

4. where there are high volumes of trucks turning onto the mainline.

46-6.03 Design Criteria

The following design criteria should be considered when designing an acceleration lane at an intersection.

1. Types. Acceleration lanes at at-grade intersections are typically the parallel design. Chapter Forty-eight provides additional information on acceleration lanes.

2. Lengths. Right-turn and median acceleration lanes should meet the criteria presented in Chapter Forty-eight. The "controlling curve" at an intersection is the design speed of the turning roadway or the speed at which a vehicle can make the right or left turn, usually less than 20 km/h. For 2-lane mainlines, the truck acceleration lengths should be considered when there are 20 to 50 or more turning trucks per day. For 4-lane facilities, they should be considered when the turning truck ADT is 75 to 100 or more vehicles.

3. <u>Taper</u>. A 90 m taper should be used at the end of the parallel acceleration lane.

46-7.0 EXTENSION OF ADDITIONAL THROUGH LANES

To meet the level-of-service criteria for the design year, it may be necessary to add through lanes approaching an intersection. However, these additional lanes should be extended beyond the intersection to fully realize the capacity benefits. Figure 46-7A, Extension of Additional Through Lanes, provides criteria for determining how far these lanes should be extended beyond the intersection.

The recommended minimum full-width lengths of the through lane extension (D_E) are those distances needed for the stopped vehicle to accelerate to 10 km/h below the average running speed of the highway. Desirably, the full-width lengths of the through lane extension will be the stored vehicle length which will cross the intersection during a green cycle.

The distances in Figure 46-7A may or may not be sufficient for the vehicle to merge into the "primary" through lane. Therefore, the criteria in Figure 46-7A should only be used for preliminary design purposes. The final design will be based on site conditions and traffic volumes and will be determined on a site-by-site basis.

The taper rate at the end of the additional through lane will be based on the criteria presented in Figure 46-7A. If curbing is used within the taper area, then the curbing should be painted to provide better delineation of the taper.

46-8.0 MEDIAN OPENINGS

<u>46-8.01 Non-Freeways</u>

46-8.01(01) Warrants

Desirably, median openings will be provided on divided non-freeways at all intersections with public roads and major traffic generators (e.g., shopping centers). However, this may result in close intersection spacing which impairs the operation of the facility. The following recommended minimum spacings should be evaluated when determining the warrant for a median opening.

1. <u>Rural Intersections</u>. Openings are generally provided at all public road intersections. Occasionally, an opening may not be provided at a minor public road.

2. <u>Urban Intersections</u>. In general, median openings are typically provided at all intersections. However, to improve capacity and traffic efficiency, the designer may elect not to provide

an opening for a traffic generator if there are other points of access within a reasonable distance of the generator.

The minimum spacing between median openings should be at least 120 m and desirably 250 m. See Figure 46-8A, Recommended Median Opening Spacing (Non-Freeway). This in general allows for the development of a future exclusive left-turn lane with the proper taper, deceleration and storage length.

When determining median opening locations, the designer also needs to consider the available sight distance (see Section 46-10). Openings with restricted sight distance may require additional design considerations (e.g., traffic signal, closing the opening).

46-8.01(02) Design

Figure 46-8B, Median Opening Design, illustrates the turning path for a semi-trailer design vehicle and other design criteria at a median opening. The following will apply.

1. Design Vehicle. An important factor in designing median openings is the path of each design vehicle making a minimum left turn at 15 to 25 km/h. Where the volume and type of vehicles making the left-turn movement call for higher than minimum speed, the design may be made by using a radius of turn corresponding to the speed deemed appropriate. However, the minimum turning path at low speed is needed for minimum design, and for testing layouts developed for one design vehicle when used by an occasional larger vehicle.

2. Radii. The following control radii may be used for minimum practical design of median ends.

    a. 12 m. Accommodates P vehicles and occasional SU vehicles with some swinging wide.

    b. 15 m. Accommodates SU vehicles and occasional WB-12 vehicles with some swinging wide.

    c. 23 m. Accommodates WB 12 and WB-15 vehicles with only minor swinging at the end of the turn.

3. Encroachment. The desirable design will be to allow the design vehicle to make the left-turn and remain entirely within the inside travel lane of the divided facility (i.e., there will be no encroachment into the through lane adjacent to the inside travel lane). It will be, however, acceptable for the design vehicle to occupy both travel lanes in its turn.

4. <u>Width</u>. In general, the median width will be determined by the design of the major highway and the available right-of-way. In addition, the designer should consider the following:

   a. If practical, the median width at the intersection should be wide enough to fully protect a stopped passenger car within the median.

   b. Slotted left-turn lanes may be used with median widths greater than 7.3 m, see Section 46-4.02(04).

   c. The median width at an intersection should be wide enough to accommodate a left-turn bay and, where necessary, a dual left-turn bay.

5. <u>Median Nose Design</u>. The most common types of median noses are the semicircular end and the bullet-nose end. The median nose design selection is dependent upon the median opening function and the median width. The *INDOT Standard Drawings* illustrate the Department's typical design criteria for median noses.

6. <u>Length of Opening</u>. The length of a median opening should properly accommodate the turning path of the design vehicle. The minimum median opening length is 12 m. It should be as great as the width of crossroad traveled way plus shoulders. If the crossroad is a divided highway, the length of opening should be at least equal to the width of the crossroad traveled way plus that of the median. However, each median opening should be evaluated individually to determine the proper length of opening. Figures 46-8C, Minimum Design of Median Openings, Design Vehicle: P, Control Radius: 12 m; Figure 46-8D, Minimum Design of Median Openings, Design Vehicle: SU, Control Radius: 15 m: and Figure 46-8E, Minimum Design of Median Openings, Design Vehicle WB-12, Control Radius: 23 m, illustrate median opening criteria for various design vehicles. The designer should consider the following factors in the evaluation.

   a. Turning Templates. The designer should check the proposed design with the turning template for the design vehicle most likely to use the intersection. Consideration should be given to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

   b. Nose Offset. At 4-leg intersections, traffic passing through the median opening (going straight) will pass the nose and the median end (semicircular or bullet nose). To provide a sense of comfort for these drivers, the offset between the nose and the through travel lane (extended) should be at least 0.6 m.

   c. Lane Alignment. The designer should ensure that lanes line up properly for crossing traffic.

d. Location of Crosswalks. Desirably, pedestrian crosswalks will intersect the median nose to provide some refuge for pedestrians. Therefore, the median opening design should be coordinated with the location of crosswalks.

In general, median openings longer than 25 m should be avoided, regardless of skew.

7. U-turns. Median openings are sometimes used only to accommodate U-turns on divided non-freeways. Where needed, the spacing should generally be 400 to 800 m. The design for U-turn maneuvers on arterials excluding Interstate routes should permit the design vehicle to turn from an auxiliary left-turn lane in the median into the lane next to the outside shoulder or outside curb and gutter on the roadway of the opposing traffic lanes. The INDOT *Standard Drawings* provide additional information on the Department's U-turn median opening design.

8. Pavement. Median opening pavements will generally be the same material type and design strength as the adjacent mainline. Chapter Fifty-two provides additional information on pavement designs.

9. Drainage. The designer should ensure that drainage from the mainline is not allowed to flow or pond within the median opening. See Part IV for INDOT roadway drainage criteria.

10. Skew. A control radius for design vehicles as the basis for the minimum design of median openings results in lengths of openings that increase with the skew angle of the intersection. The skew introduces other variations in the shape of the nose. At a skewed crossing, such control radius should be used in the acute angle used to locate the beginning of the nose on the median edge.

    Special channelization, left-turn lanes, or adjustments to reduce the crossroad skew may be required to limit the opening to the maximum length noted in Item 6 above. In general, an asymmetrical bullet nose is preferable.

46-8.02 Median Openings on Freeways

On fully access-controlled freeways, median crossings are denied to the public. However, occasional median openings or emergency crossovers are required to accommodate maintenance and emergency vehicles. Section 54-6.0 provides the Department's criteria for median openings on fully access-controlled facilities.

46-9.0 CHANNELIZING ISLANDS

Several of the treatments described in this chapter require channelizing islands within the intersection area (e.g., turning roadways). The design of islands should consider several site-specific functions, including definition of vehicular paths, separation of traffic movements, prohibition of movements, protection of pedestrians and placement of traffic control devices.

46-9.01 Types of Islands

Islands can be grouped into the following functional classes. Most islands serve at least two of these functions.

1. Directional Islands. Directional islands (e.g., for turning roadways) control and direct traffic movements and guide the driver into the proper channel.

2. Divisional Islands. Divisional islands separate opposing traffic flows, alert the driver to the crossroad ahead and regulate traffic through the intersection. These islands are often introduced at intersections on undivided highways and are particularly advantageous in controlling left turns at skewed intersections.

3. Refuge Islands. Refuge islands at or near crosswalks aid or protect pedestrians crossing a wide roadway. These islands may be required for pedestrians at intersections where complex signal phasing is used.

4. Protection Islands. Protection islands are often used for the installation of traffic control devices.

46-9.02 Selection of Island Type

Channelizing islands may be some combination of flush or raised, concrete/asphalt or turf, and triangular or elongated. Selection of an appropriate type of traffic island should be based on traffic characteristics, cost considerations and maintenance needs. The road designer will be responsible for the selection and design of the channelizing island. The following offers guidance as to when flush or raised corrugated islands are appropriate.

Flush islands are appropriate as follows:

1. on high-speed rural highways to delineate separate turning lanes;

2. in constrained locations where vehicular path definition is desired, but space for larger, raised islands is not available;

3. to separate opposing traffic streams on low-speed streets; and/or

4. for temporary channelization either during construction or to test traffic operations prior to installation of raised islands.

Raised corrugated islands are appropriate as follows:

1. where a primary function of the island is to provide a pedestrian refuge;

2. where a primary or secondary island function is the location of traffic signals, signs or other fixed objects;

3. where the island is intended to prohibit or prevent traffic movements;

4. on low- to moderate-speed highways where the primary function is to separate high volumes of opposing traffic movements; and/or

5. at locations requiring positive delineation of vehicular paths, such as at major route turns or intersections with unusual geometry.

Channelizing islands with curbs should not be used.

46-9.03 Minimum Size

Traffic islands should be large enough to command the driver's attention. Island shapes and sizes vary from one intersection to another. For triangular islands, the recommended minimum size is 5 m^2 (urban) and 7 m^2 (rural). Desirably, triangular islands will be at least 10 m^2. Islands used for pedestrian refuge should be at least 15 m^2. Elongated islands should not be less than 1.2 m wide, and should be 6 to 8 m long. Where space is limited, enlongated islands may be reduced to a minimum width of 0.6 m. Curbed divisional islands introduced at isolated intersections on high-speed highways should be at least 30 m in length.

46-9.04 Delineation

Delineation of small islands is effected primarily by curbs. Large curbed islands may be sufficiently delineated by color and texture contrast of vegetative cover, mounded earth, shrubs, reflector posts, or any combination of these. In rural areas, island curbs should usually be sloping. Vertical or sloping curbs may be appropriate in urban areas, depending on the conditions.

Channelizing islands should be delineated based on their size, location and function. Islands with raised corrugations present the most positive means of delineation and may be used with all design speeds. For the design of raised corrugated islands, the designer is referred to the *INDOT Standard Drawings*. For flush islands, it may be appropriate to complement the pavement markings with raised reflectors.

Raised pavement markings, raised reflectors, roughened pavement and/or paint striping is used in advance of and around the island to warn the driver. These traffic control devices are especially important at the approach to divisional curbed islands for the direction of approaching traffic. Figure 46-9A, Triangular Island, and Figure 46-9B, Elongated Islands, illustrate the typical pavement markings used with channelizing islands. Section 76-2.03 provides additional information for pavement markings around islands.

46-9.05 Island Offset to Through Lanes

In urban areas on approach roadways without shoulders, the raised corrugated island should be offset 0.6 m from the travel lane. Where shoulders are present, the raised corrugated island should be offset a distance equal to the shoulder width. In rural areas and where separate turning lanes are used, the island should be offset from the turning lane by 0.6 m (see Figure 46-9A). If there are no turning lanes, the island should be offset a distance equal to the shoulder width. If the corner island is preceded by a right-turn deceleration lane, the shoulder offset should be at least 2.4 m.

The designer should also ensure that the island will not interfere with the turning movement of a truck turning from the opposite side on a 4-legged intersection. If there is a conflict, the island should be set back farther or made flush.

46-9.06 Typical Channelizing Intersections

Figure 46-9C, Example of a Channelizing Intersection, illustrates an example of an island treatment. Each channelizing intersection must be studied individually considering turning volumes, traffic lane configurations, potential conflicts and practical signing arrangements.

46-10.0 INTERSECTION SIGHT DISTANCE (ISD)

For an at-grade intersection to operate properly, adequate sight distance should be available. The designer should provide sufficient sight distance for a driver to perceive potential conflicts and to perform the actions needed to negotiate the intersection safely.

The additional costs and impacts of removing sight obstructions are often justified. If it is impractical to remove an obstruction blocking the sight distance, the designer should consider providing traffic control devices or design applications (e.g., warning signs, traffic signals or turn lanes) which may not otherwise be warranted.

The height of eye for a passenger car driver should be taken as 1080 mm. The height of eye for a single unit or combination truck driver should be taken as 2330 mm. Its height of object should be taken as 1080 mm.

The sight line is shown on the plans in the plan and profile views. The proposed profile grade line along the centerline is also shown, however, this is meaningless for intersection sight distance analysis. The proposed ground line under the sight line is the relevant line.

> ** *PRACTICE POINTER* **
>
> Intersection sight distance should be analyzed for each local service road or frontage road in the same manner as a public road.

46-10.01 No Traffic Control

Intersections between low-volume and low-speed roads/streets should be either yield controlled or stop controlled. However, for county/city roads and streets intersections with no traffic control, sufficient corner sight distance should be available to allow approaching vehicles to see potentially conflicting vehicles in sufficient time to stop before reaching the intersection. Figure 46-10A, Intersection Sight Distance (No Traffic Control), provides the ISD criteria with approach grades between –3% and +3%. For approach grades greater than 3%, multiply the sight distance value in Figure 46-10A by the appropriate adjustment factor from Figure 46-10B, Adjustment Factors for Sight Distance with No Traffic Control. These figures are not applicable for State highways.

If the appropriate sight distance cannot be provided, consideration should be given to installing stop signs on one or more approaches.

46-10.02 Yield Control

46-10.02(01) Intersections With Yield Control on the Minor Road

Drivers approaching yield signs are permitted to enter or cross the major road without stopping, if there are no potentially conflicting vehicles on the major road. The sight distances needed by drivers on yield-controlled approaches exceed those for stop-controlled approaches.

Yield-controlled approaches generally need greater sight distance than stop-controlled approaches, especially at four-leg yield-controlled intersections where the sight distance needs of the crossing maneuver should be considered. If sight distance sufficient for yield control is not available, use of a “Stop” sign instead of a “Yield” sign should be considered. In addition, at locations where the recommended sight distance cannot be provided, consideration should be given to installing other traffic control devices at the intersection on the major road to reduce the speeds of approaching vehicles.

46-10.02(02) Left- and Right-Turn Maneuvers

The length of the leg of the approach sight triangle along the minor road to accommodate left and right turns without stopping should be 25 m. This distance is based on the assumption that drivers making left and right turns without stopping will slow to a turning speed of 16 km/h.

The leg of the approach sight triangle along the major road is similar to the major-road leg of the departure sight triangle for a stop-controlled intersection. However, the time gaps for a left turn, as shown in Section 46-10.03 should be increased by 0.5 s to the values shown in Figure 46-10C, Time Gaps for Left or Right Turns, Yield Control. The appropriate lengths of the sight triangle leg are shown in Figure 46-10D, Design Intersection Sight Distance, Left or Right Turn at Yield-Controlled Intersection, for passenger cars. The minor-road vehicle needs 3.5 s to travel from the decision point to the intersection. This represents additional travel time that is needed at a yield-controlled intersection, but is not needed at a stop-controlled intersection. However, the acceleration time after entering the major road is 3.0 s less for a yield sign than for a stop sign because the turning vehicle accelerates from 16 km/h rather than from a stop condition. The net 0.5-s increase in travel time for a vehicle turning from a yield-controlled approach is the difference between the 3.5-s increase in travel time and the 3.0-s reduction in travel time.

Departure sight triangles like those provided for stop-controlled approaches should also be provided for yield-controlled approaches to accommodate minor-road vehicles that stop at the yield sign to avoid conflicts with major-road vehicles. However, because approach sight triangles for turning maneuvers at yield-controlled approaches are larger than the departure sight triangles used at stop-controlled intersections, no specific check of departure sight triangles at yield-controlled intersection should be needed.

Yield-controlled approaches generally need greater sight distance than stop-controlled approaches, especially at four-leg yield-controlled intersections where the sight distance needs of the crossing maneuver should be considered. If sight distance sufficient for yield control is not available, use of a stop sign instead of a yield sign should be considered. In addition, at locations where the recommended sight distance cannot be provided, consideration should be given to installing other traffic control devices at the intersection on the major road to reduce the speeds of approaching vehicles.

46-10.02(03) Turning Roadways

Yield control may also exist, for example, at a freeway ramp terminal where the ramp traffic is provided a free-flowing right turn onto the minor road. The assumptions as discussed in Section 46-10.02(01) are also applicable to turning roadway yield conditions except the eye location of the entering vehicle is typically on the turning roadway itself (see Figure 46-10E, Intersection Sight Distance for Turning Roadways).

If insufficient intersection sight distance is available for the operational characteristics of yield control, it may be appropriate to convert the intersection to a stop control.

46-10.03 Stop Control

Where traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position assuming that the approaching vehicle comes into view as the stopped vehicle begins its departure. The location of the eye should be 5.4 m from the edge of the travel lane.

46-10.03(01) Departure Sight Triangles and Time Gaps

Departure sight triangles for intersections with stop control on the minor road must consider three situations as follows:

1. left turns from the minor road
2. right turns from the minor road
3. crossing the major route from minor road approach.

Departure sight triangles for traffic approaching from either the right or left, like those shown in Figure 46-10F, Departure Sight Triangles, should be provided for left turns from the minor road onto the major road for all stop-controlled approaches.

Field observations of the gaps in major-road traffic actually accepted by drivers turning onto the major route have shown that the values shown in Figure 46-10G, Intersection Sight Distance for Stop-Controlled Intersection, provide sufficient time for the minor-road vehicle to accelerate from a stop and complete a left turn without unduly interfering with major-road traffic operations.

The intersection sight distance in both directions should be equal to the distance traveled at the design speed of the major road during a period of time equal to the time gap. At a minimum, the designer should check ISD for both a passenger car and a single unit truck turning from the minor-road approach. Where substantial volumes of heavy vehicles enter the major road, the use of combination trucks should be considered.

Generally, no adjustment is needed for the major-road grade; however, if the minor-road design vehicle is a truck and the intersection is located near a sag vertical curve with a grade over 3%, an adjustment of the intersection sight distance should be considered.

Figure 46-10G provides the criteria for intersection sight distance in both directions for vehicles turning left.

Intersection sight distance for left turns at divided highway intersections should consider multiple design vehicles and median width. If the design vehicle used to determine sight distance for a divided highway intersection is larger than a passenger car, sight distance for left turns will need to be checked for that selected design vehicle and for smaller design vehicles as well. If the divided highway median is wide enough to store the design vehicle with a clearance to the through lanes of 1 m at both ends of the vehicle, no separate analysis for the departure sight triangle for left turns is needed on the minor-road approach for the near roadway to the left.

If the design vehicle can be stored in the median with adequate clearance to the through lanes, a departure sight triangle to the right for left turns should be provided for that design vehicle turning left from the median roadway. Where the median is not wide enough to store the design vehicle, a departure sight triangle should be provided for that design vehicle to turn left from the minor-road approach. The median width should be considered in determining the number of lanes to be crossed. The median width should be converted to an equivalent number of lanes.

In most cases, the sight triangles for left and right turns onto the major road will also provide more than adequate sight distance for minor-road vehicles to cross the major road. However, in the following situations intersection sight distance for crossing maneuvers must be checked:

1. Where left and/or right turns are not permitted from a particular approach and the crossing maneuver is the only legal maneuver;
2. Where the crossing vehicle would cross the equivalent width of more than 6 lanes; or
3. Where substantial volumes of heavy vehicles cross the highway and steep grades that might slow the vehicles while its back portion is still in the intersection are present on the departure roadway on the far side of the intersection.

The time gaps shown in Figure 46-10H_1, Time Gaps for Crossing Maneuvers, may be used for crossing maneuvers.

Figure 46-10H, Intersection Sight Distance for Passenger Car to Turn Right) provides the intersection sight distance for a passenger car making a right turn from a stop or a crossing maneuver.

At divided highway intersections, depending on the median width and the length of the design vehicle, intersection sight distance may need to be considered for crossing both roadways of the divided highway or for crossing the near lanes only and stopping in the median before proceeding.

The ISD values will establish one leg of the sight triangle which needs to be visible to the entering vehicle. The leg on the stop-controlled road or street will be determined by the assumed location of the eye. This is established as 5.4 m behind the edge of the travel lane on new or reconstruction projects and 4.4 m on 3R projects (see Figure 46-10F, Departure Sight Triangles).

46-10.03(02) Measures to Improve Intersection Sight Distance

The available ISD should be checked using the above noted parameters. If the line of sight falls above the bridge railing and guardrail and the ISD value from Figure 46-10G, Intersection Sight Distance for Stop-Controlled Intersection, is provided, no further investigation is needed. If the line of sight is restricted by either the bridge railing, guardrail, other obstructions, or the horizontal and vertical alignment of the major road and the ISD value is not available, one or more of the following modifications, or a combination of them, should be evaluated to achieve the intersection sight distance:

1. relocate the minor road or drive farther from the end of the bridge,

2. widen the structure on the side where the railing is restricting the line of sight,
3. flare the approach guardrail,
4. revise the grades on the major road and/or the minor road or drive,
5. remove the obstructions that are restricting sight distance,
6. close the minor road or drive,
7. make the minor road or drive one-way away from the major road, and/or
8. review other measures that may be practical at a particular location.

If intersection sight distance along the major road cannot be achieved, consideration should be given to installing advance intersection signing with advisory speed plates.

46-10.04 Left Turns From the Major Road

All locations along the major road from which vehicles are permitted to turn left across opposing traffic, including intersections and driveways, should have sufficient sight distance to accommodate the left-turn maneuver. Left-turning drivers need sufficient sight distance to decide when it is safe to turn left across the lane(s) used by opposing traffic. Sight distance design should be based on a left turn by a stopped vehicle, since a vehicle that turns left without stopping would need less sight distance. The sight distance along the major road to accommodate left turns is the distance traversed at the design speed of the major road in the travel time for the design vehicle given in Figure 46-10 I, Time Gaps for Left Turn from the Major Road.

The table also contains appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle. The unadjusted time gap in Figure 46-10 I for passenger cars was used to develop the sight distances in Figure 46-10J, Intersection Sight Distance for Left Turn from the Major Road.

If stopping sight distance has been provided continuously along the major road and if sight distance for stop control or yield control has been provided for each minor-road approach, sight distance will generally be adequate for left turns from the major road.

However, at three-leg intersections located on or near a horizontal curve or crest vertical curve on the major road, the availability of adequate sight distance for left turns from the major road should be checked. In addition, the availability of sight distance for left turns from a divided highway should be checked because of the possibility of sight obstructions in the median.

At 4-leg intersections on divided highways, opposing vehicles turning left can block a driver’s view of oncoming traffic. The designer should consider offsetting the opposing left-turn lanes and providing left-turning drivers with a better view of oncoming traffic.

46-10.05 Signal-Controlled Intersections

If a vehicle is allowed to turn right on red (or left from a one-way street onto a one-way street) after stopping, the minimum ISD requirements in Figure 46-10H, Intersection Sight Distance for Passenger Car to Turn Right), will apply to a signalized intersection. If these criteria cannot be met, consideration should be given to prohibiting right-turn-on-red at the intersection. This determination will be based on field investigations and will be determined on a case-by-case basis. Changing right-turn-on-red regulations at an intersection will require an official action by State and/or local officials.

If the signal is to be placed on two-way flashing operation (i.e., flashing yellow on the major-road approaches and flashing red on the minor-road approaches) during off-peak or nighttime conditions, the appropriate departure sight triangles for the stop control, both to the left and to the right, should be provided for the minor-road approaches (See Section 46-10.03).

46-10.06 Effect of Skew

Where two highways intersect at an angle of less than 60 deg, some of the factors for determination of intersection sight distance may need adjustment.

Each of the clear sight triangles described above are applicable to oblique-angle intersections. As shown in Figure 46-10K, Sight Triangles at Skewed Intersections, the legs of the sight triangle will lie along the intersection approaches, and each sight triangle will be larger or smaller than the corresponding sight triangle would be at a right-angle intersection. The area within each sight triangle should be clear of potential sight obstructions.

At an oblique-angle intersection, the lengths of the travel paths for some turning and crossing maneuvers will be increased. The actual path length for a turning or crossing maneuver may be computed by dividing the total widths of the lanes (plus the median width, where appropriate) to be crossed by the sine of the intersection angle. If the actual path length exceeds the total widths of the lanes to be crossed by 3.6 m or more, an appropriate number of additional lanes should be considered in applying the adjustment for the number of lanes to be crossed (See Section 46-10.03). For a crossing maneuver from a minor road with yield control, the w term in the equation for the major-route leg of the sight triangle to accommodate the crossing maneuver should also be divided by the sine of the intersection angle to obtain the actual path length. In the obtuse-angle quadrant of an oblique-angle intersection, the angle between the approach leg and the sight line is often so small that drivers can look across the full sight triangle with only a small head movement. However, in the acute-angle quadrant, drivers are often required to turn their heads considerably to see across the entire clear sight triangle. For this reason, it is recommended the sight distance criteria for intersections with no control not be applied to oblique-angle intersections and that sight distances at least equal to those for intersections with stop control on the minor road should be provided, where practical.

46-11.0 DRIVEWAY DESIGN

46-11.01 General Information

46-11.01(01) Definitions of Drives and Types

The definitions of types and classes of drives are as follows:

1. Residential. A residential drive provides access to a single family residence, duplex, or apartment building with not more than four dwelling units. A residential drive along a roadway with a raised curb is a class I drive. A residential drive along a roadway with a paved or unpaved shoulder and no raised curb is a class II drive.

2. Commercial. A commercial drive provides access to an office, retail, or institutional building, or to an apartment building with five or more dwelling units. A drive which serves an industrial plant, but with a primary function to serve an administrators' or employees' parking lot, is considered to be a commercial drive. A commercial drive along a roadway with a raised curb is a class III drive. A commercial drive along a roadway with a paved or unpaved shoulder and no raised curb is a class IV drive.

3. Industrial. An industrial drive directly serves substantial numbers of truck movements to and from loading docks of an industrial facility, warehouse, or truck terminal. A centralized retail development, such as a community or regional shopping center, may have one or more drives especially so designed, signed, and located to provide access for trucks. This is also classified as an industrial drive. An industrial drive may be designed either as a public road approach or as an industrial drive. An industrial drive along a roadway with a raised curb is a class VII drive. An industrial drive along a roadway with a paved or unpaved shoulder and no raised curb is a class VI drive.

4. Field Entrance. A field entrance provides access to an unimproved property, e.g., a farm field with no buildings. Such a drive along a roadway with a paved or unpaved shoulder is a class V drive.

46-11.01(02) Drive Spacing and Corner Clearances

Closely-spaced drives can cause operational problems, especially with a high-volume roadway and/or high-volume drives. These problems can also result if a drive is too close to an at-grade intersection.

Desirably, any part of a drive, including its entrance radius, should not be placed within the radius of a public road at an intersection, including any auxiliary lanes. Preferably, there should be a 6- to 12-m tangent section between the drive radius and the public road radius for greater separation. If this criterion cannot be met for a property at an intersection corner, one solution may be to relocate the drive entrance from the major road to the minor road, if practical. Another possible solution is to provide a right-turn lane at the intersection. This will improve the operation of the intersection by removing the turning vehicles for the drive and intersection out of the through travel lane(s). However, significant numbers of turning vehicles may impair egress from the property.

Drives for the same owner should be located across from each other (e.g., a farm) where crossing traffic is significant or where it is not desirable to permit slow or large equipment to travel along the highway or shoulder.

46-11.01(03) Drive Sight Distance

Section 46-10.0 discusses intersection sight distance (ISD) criteria for an intersection with a public road. Desirably, these criteria will also apply to sight distance at a drive. However, for a drives with low traffic volume, it is not warranted to explore extraordinary measures to improve sight distance. Sight obstructions, e.g., large trees, hedgerows, etc., should be checked for in the vicinity of the drive entrance which may limit sight distance. To perform the check, it is reasonable to assume an eye location of approximately 3 m from the edge of travel lane.

If drive sight-distance criteria with the eye location described above cannot be met, informal notification should be provided to the project reviewer for a consultant-designed project or to the supervisor for an in-house project.

46-11.01(04) Auxiliary Lanes

Deceleration and acceleration lanes should be considered at each high-volume drive entrance, especially on a high-speed, high-volume arterial. Sections 46-4.0 and 46-7.0 further discuss the design and warrants for these auxiliary lanes, which may also apply to a high-volume drive. In addition to traffic-volume considerations, it may be warranted to provide a right-turn lane into the drive if the change in grade is abrupt at the drive entrance.

46-11.01(05) Joint Residential or Commercial Drive

If practical and agreeable to the property owners, the use of a joint drive offers one option to reduce the number of access points along the highway. The centerline of the joint drive should be located on the property line dividing the two owners. This practice will not allow either owner the opportunity to deny or restrict access to the neighbor's property and, depending on the traffic volume, may improve the traffic flow on the mainline. For a commercial drive, this may require providing a drive wide enough to handle two-way traffic.

46-11.02 Design Criteria

The INDOT *Standard Drawings* provide the Department's design criteria for the various drive classes. In addition, the following should be considered.

46-11.02(01) Class Determination Considerations

1. If it is determined from the survey or at the field inspection that a field entrance serves a barn or storage shed for farm machinery, it should be designed as a class II drive with a 7.2 m minimum width instead of a class V drive.

2. Where there are positive indications that a private residence is being used for commercial purposes, the drive should be designed as a commercial drive.

46-11.02(02) Radii

1. Class II and class IV drive radii should start from the edge of the paved shoulder if the width of the paved shoulder is 2.4 m or greater.

2. Class II and class IV drive radii should start from the edge of the traveled way if the width of the paved shoulder is less than 2.4 m.

3. Class VI drive tapers should start from the edge of the traveled way without regard to the shoulder's width or whether or not the shoulder is paved.

46-11.02(03) Width

1. Drive width should be measured perpendicular to the centerline of the drive.

2. For each new drive constructed where no drive currently exists, the minimum width shown on the INDOT *Standard Drawings* should be used, unless determined otherwise at the field inspection or if the Land Acquisition Division recommends a wider width.

3. The width of a reconstructed drive should be the same as the existing width but not less than the minimum width nor greater than the maximum width shown on the INDOT *Standard Drawings*.

4. Each drive that serves a barn or storage shed for farm equipment should be a minimum of 7.2 m in width.

46-11.02(04) Drive Grades

For a class I, III, VI, or VII drive, the maximum algebraic difference in drive grades should not exceed 8% for a crest vertical curve, or 12% for a sag vertical curve. For a class II, IV, or V drive, the maximum algebraic difference in drive grades should not exceed 11% for a crest vertical curve, or 14% for a sag vertical curve.

If it is known that large emergency vehicles or other large vehicles will be using a drive, or if the algebraic differences exceed those noted above, the fit of the drive grade should be checked against the vehicle templates.

Drive grades should be shown and drive PVIs should be identified on the cross-sections sheets.

46-11.02(05) Grading

The drive's embankment slope within the mainline clear zone should be as shown in Figure 05-31B, Drive Embankment Slopes Within Clear Zone. Outside the clear zone, the embankment slope should desirably be 4:1, but should not be steeper than 3:1.

46-11.02(06) Paving

1. Each residential, commercial, or industrial drive should have either an asphalt or concrete surface as shown on the INDOT *Standard Drawings* from the edge of the mainline pavement to at least the highway right-of-way line. The drive pavement should be replaced in kind beyond the right-of-way line only if required to match grade or alignment, and not to repair the drive due to condition.

2. A field entrance typically has an unimproved soil surface within the right-of-way, except as discussed in Section 46-11.02(01) Item 1.

46-11.02(07) Intersecting Sidewalk Treatment

1. Sidewalk curb ramps should only be used with a signalized class III or class VII drive.

2. For a class I drive or nonsignalized class III or class VII drive, a sidewalk elevation transition as shown on the INDOT *Standard Drawings* should be used.

46-11.03 Impacts to Project with Drive Designs Complete and Right of Way Acquisition Under Way

Each Class I or III drive should have its grade designed in accordance with the INDOT *Standard Drawings*. However, if the profile-grade requirements shown in such *Standard Drawings* extend an already-designed drive outside the available right of way, such drive should have its grade detailed on the plans so that the drive remains inside the available right of way. Such drive should also be checked for accessibility by large emergency vehicles or other large vehicles. Such drive should be identified as modified.

46-12.0 TURNING TEMPLATES

Figures 46-12A through 46-12P provide turning templates for the design vehicles most commonly used by INDOT. Turning templates are included for the design vehicles listed below in scales of 1:200 and 1:500.

1.	P	Passenger car, light panel truck, or pickup truck
2.	SU	Single-unit truck
3.	S-BUS-11	Conventional school bus (65 passengers)
4.	WB-12	Intermediate semitrailer combination
5.	WB-15	Intermediate semitrailer combination
6.	WB-19	Interstate semitrailer combination
7.	WB-20 (IDV)	Indiana Design Vehicle: Interstate semitrailer combination
8.	WB-33D	Turnpike semitrailer combination with two trailers
9.	MH/B	Recreational vehicle: motor home and boat trailer

The list of the figures is as follows:

46-12A P - Design Vehicle (1:200 Scale)

46-12B	P - Design Vehicle (1:500 Scale)
46-12C	SU - Design Vehicle (1:200 Scale)
46-12D	SU - Design Vehicle (1:500 Scale)
46-12D(1)	S-BUS-11 - Design Vehicle (1:300 Scale)
46-12D(2)	S-BUS-11 - Design Vehicle (1:500 Scale)
46-12E	WB-12 - Design Vehicle (1:300 Scale)
46-12F	WB-12 - Design Vehicle (1:500 Scale)
46-12G	Figure Deleted
46-12H	WB-15 - Design Vehicle (1:500 Scale)
46-12 I	Figure Deleted
46-12J	WB-19 - Design Vehicle (1:500 Scale)
46-12K	Figure Deleted
46-12L	WB-20 (IDV) - Design Vehicle (1:500 Scale)
46-12M	Figure Deleted
46-12N	WB-33D - Design Vehicle (1:500 Scale)
46-12 O	MH/B - Design Vehicle (1:300 Scale)
46-12P	MH/B - Design Vehicle (1:500 Scale)

The figures show the turning paths for the above listed AASHTO design vehicles. The paths shown are for the left-front overhang and outside-rear wheel. The left-front wheel follows the circular curve; however, its path is not shown.